7. DESIGN

7.1 Frame Design

The best way to increase a structure's likelihood of responding to seismic attack in its fundamental mode of vibration is to balance its stiffness and mass distribution. Irregularities in geometry increase the likelihood of complex nonlinear response that cannot be accurately predicted by elastic modeling or plane frame inelastic static modeling.

7.1.1 Balanced Stiffness

It is strongly recommended that the ratio of effective stiffness between any two bents within a frame or between any two columns within a bent satisfy equation 7.1. It is strongly recommended that the ratio of effective stiffness between adjacent bents within a frame or between adjacent columns within a bent satisfy equation 7.2. An increase in superstructure mass along the length of the frame should be accompanied by a reasonable increase in column stiffness. For variable width frames the tributary mass supported by each bent or column shall be included in the stiffness comparisons as specified by equation 7.1(b) and 7.2(b). The simplified analytical technique for calculating frame capacity described in Section 5.5 is only permitted if either 7.1(a) & 7.2(a) or 7.1(b) & 7.1(b) are satisfied.

Constant Width Frames

$$k_i^e / k_j^e \geq 0.5 \qquad (7.1a)$$

$$k_i^e / k_j^e \geq 0.75 \qquad (7.2a)$$

k_i^e = The smaller effective bent or column stiffness

k_j^e = The larger effective bent or column stiffness

Variable Width Frames

$$\frac{k_i^e}{m_i} \Big/ \frac{k_j^e}{m_j} \geq 0.5 \qquad (7.1b)$$

$$\frac{k_i^e}{m_i} \Big/ \frac{k_j^e}{m_j} \geq 0.75 \qquad (7.2b)$$

m_i = Tributary mass of column or bent i

m_j = Tributary mass of column or bent j

The following considerations shall be taken into account when calculating effective stiffness: framing effects, end conditions, column height, percentage of longitudinal and transverse column steel, column diameter, and foundation flexibility. Some of the consequences of not meeting the relative stiffness recommendations defined by equations 7.1 and 7.2 include:

- Increased damage in the stiffer elements
- An unbalanced distribution of inelastic response throughout the structure
- Increased column torsion generated by rigid body rotation of the superstructure

7.1.2 Balanced Frame Geometry

It is strongly recommend that the ratio of fundamental periods of vibration for adjacent frames in the longitudinal and transverse direction satisfy equation 7.3.

$$T_i / T_j \geq 0.7 \tag{7.3}$$

T_i = Natural period of the less flexible frame

T_j = Natural period of the more flexible frame

The consequences of not meeting the fundamental period requirements of equation 7.3 include a greater likelihood of out-of-phase response between adjacent frames leading to large relative displacements that increase the probability of longitudinal unseating and collision between frames at the expansion joints. The colliding and relative transverse translation of adjacent frames will transfer the seismic demand from one frame to the next, which can be detrimental to the stand-alone capacity of the frame receiving the additional seismic demand.

7.1.3 Adjusting Dynamic Characteristics

The following list of techniques should be considered for adjusting the fundamental period of vibration and/or stiffness to satisfy equations 7.1, 7.2 and 7.3. Refer to Memo to Designer 6-1 for additional information on optimizing performance of bridge frames.

- Oversized pile shafts
- Adjust effective column lengths (i.e. lower footings, isolation casing)
- Modified end fixities
- Reduce/redistribute superstructure mass
- Vary the column cross section and longitudinal reinforcement ratios
- Add or relocate columns
- Modify the hinge/expansion joint layout
- Incorporate isolation bearings or dampers

A careful evaluation of the local ductility demands and capacities is required if project constraints make it impractical to satisfy the stiffness and structure period requirements in equations 7.1, 7.2, and 7.3.

7.1.4 End Span Considerations

The influence of the superstructure on the transverse stiffness of columns near the abutment, particularly when calculating shear demand, shall be considered.

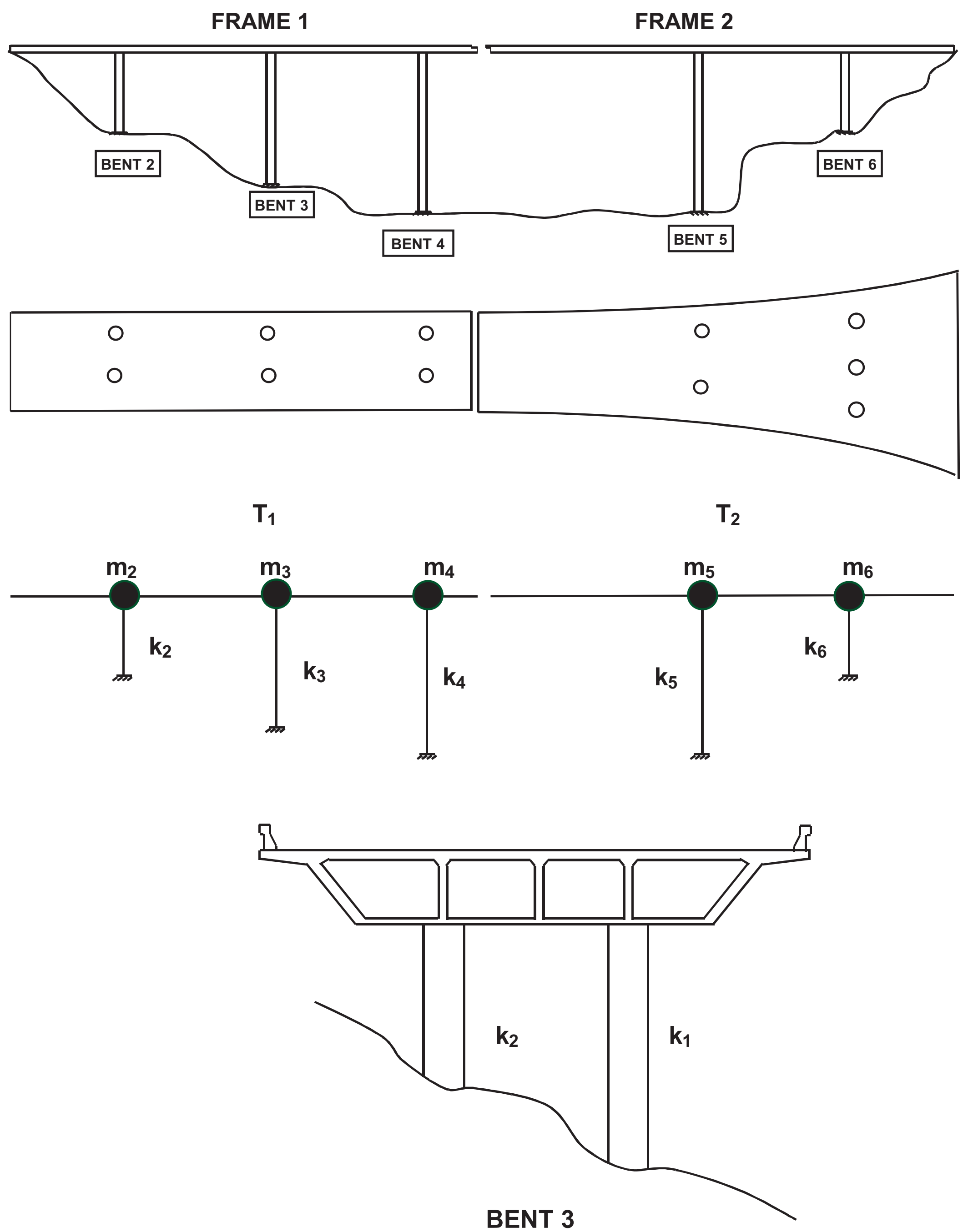

Figure 7.1 Balanced Stiffness

7.2 Superstructure

7.2.1 Girders

7.2.1.1 Effective Superstructure Width

The effective width of superstructure resisting longitudinal seismic moments is defined by equation 7.4. The effective width for open soffit structures (e.g. T-Beams & I- Girders) is reduced because they offer less resistance to the torsional rotation of the bent cap. The effective superstructure width can be increased at a 45° angle as you move away from the bent cap until the full section becomes effective. On skewed bridges, the effective width shall be projected normal to the girders where the centerline of girder intersects the face of the bent cap. See Figure 7.2.

$$B_{eff} = \begin{cases} D_c + 2 \times D_s & \text{Box girders \& solid superstructures} \\ D_c + D_s & \text{Open soffit superstructures} \end{cases} \tag{7.4}$$

Additional superstructure width can be considered effective if the designer verifies the torsional capacity of the cap can distribute the rotational demands beyond the effective width stated in equation 7.4.

If the effective width cannot accommodate enough steel to satisfy the overstrength requirements of Section 4.3.1, the following actions may be taken:

- Thicken the soffit and/or deck slabs
- Increase the resisting section by widening the column*
- Haunch the superstructure
- Add additional columns

* The benefit of using wider columns must be carefully weighed against the increased joint shear demands and larger plastic hinging capacity.

Isolated or lightly reinforced flares shall be ignored when calculating the effective superstructure width. See Section 7.6.5 for additional information on flare design.

7.2.2 Vertical Acceleration

If vertical acceleration is considered, per Section 2.1, a separate analysis of the superstructure's nominal capacity shall be performed based on a uniformly applied vertical force equal to 25% of the dead load applied upward and downward, see Figure 7.3. The superstructure at seat type abutment is assumed to be pinned in the vertical direction, up or down. The superstructure flexural capacity shall be based only on continuous mild reinforcement distributed evenly between the top and bottom slabs. The effects of dead load, primary and secondary prestressing shall be ignored. The continuous steel shall be spliced with "service level" couplers as defined in Section 8.1.3, and is considered effective in offsetting the mild reinforcement required for other load cases. Lap splices equal to two times the standard lap may be substituted for the "service splices", provided the laps are placed away from the critical zones (mid-spans and near supports).

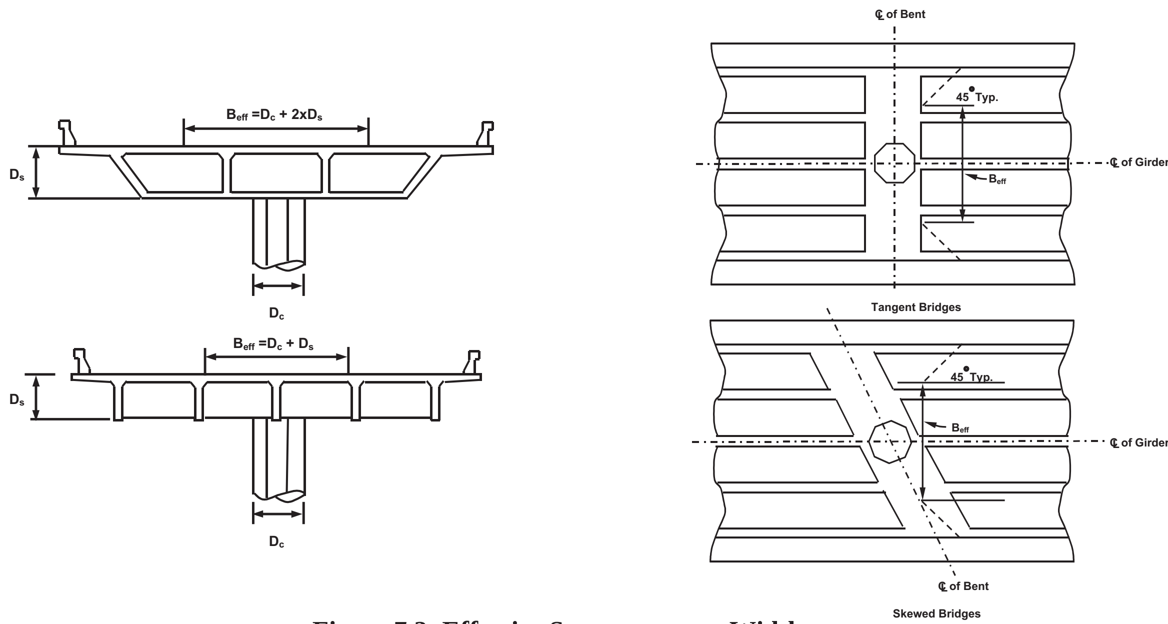

Figure 7.2 Effective Superstructure Width

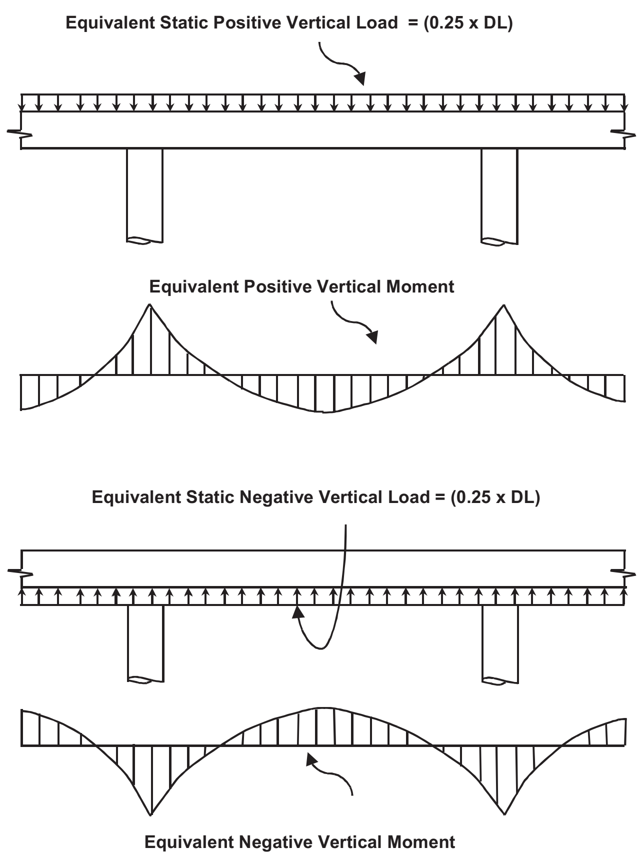

Figure 7.3 Equivalent Static Vertical Loads & Moments

The longitudinal side reinforcement in the girders, if vertical acceleration is considered per Section 2.1, shall be capable of resisting 125% of the dead load shear at the bent face by means of shear friction. The enhanced side reinforcement shall extend continuously for a minimum of 2.5 D_s beyond the face of the bent cap.

7.2.3 Pre-cast Girders

Historically precast girders lacked a direct positive moment connection between the girders and the cap beam, which could potentially degrade to a pinned connection in the longitudinal direction under seismic demands. Therefore, to provide stability under longitudinal seismic demands, columns shall be fixed at the base unless an integral girder/cap beam connection is provided that is capable of resisting the column overstrength demands as outlined in Sections 4.3.1, 4.3.2, & 7.2.2. Recent research has confirmed the viability of pre-cast spliced girders with integral column/ superstructure details that effectively resist longitudinal seismic loads. This type of system is considered non-standard until design details and procedures are formally adopted. In the interim, project specific design criteria shall be developed per MTD 20-11.

7.2.4 Slab Bridges

Slab bridges shall be designed to meet all the strength and ductility requirements as specified in the SDC.

7.2.5 Hinges

7.2.5.1 Longitudinal Hinge Performance

Intermediate hinges are necessary for accommodating longitudinal expansion and contraction resulting from prestress shortening, creep, shrinkage and temperature variations. The hinge allows each frame to vibrate independently during an earthquake. Large relative displacements can develop if the vibrations of the frames are out-of-phase. Sufficient seat width must be provided to prevent unseating.

7.2.5.2 Transverse Hinge Performance

Typically hinges are expected to transmit the lateral shear forces generated by small earthquakes and service loads. Determining the earthquake force demand on shear keys is difficult since the magnitude is dependent on how much relative displacement occurs between the frames. Forces generated with EDA should not be used to size shear keys. EDA overestimates the resistance provided by the bents and may predict force demands on the shear keys that differ significantly from the actual forces.

7.2.5.3 Frames Meeting the Requirements of Section 7.1.2

All frames including balanced frames or frames with small differences in mass and/or stiffness will exhibit some out-of-phase response. The objective of meeting the fundamental period recommendations between adjacent frames presented in Section 7.1.2 is to reduce the relative displacements and associated force demands attributed to out-of-phase response.

Longitudinal Requirements

For frames adhering to Section 7.1.2 and expected to be exposed to synchronous ground motion, the minimum longitudinal hinge seat width between adjacent frames shall be determined by Section 7.2.5.4.

Transverse Requirements

The shear key shall be capable of transferring the shear between adjacent frames if the shear transfer mechanism is included in the demand assessment. The upper bound for the transverse shear demand at the hinge can be estimated by the sum of the overstrength shear capacity of all the columns in the weaker frame. The shear keys must have adequate capacity to meet the demands imposed by service loads.

An adequate gap shall be provided around the shear keys to eliminate binding of the hinge under service operation and to ensure lateral rotation will occur thereby minimizing moment transfer across the expansion joint.

Although large relative displacements are not anticipated for frames with similar periods exposed to synchronous ground motion, certain structural configurations may be susceptible to lateral instability if the transverse shear keys completely fail. Particularly skewed bridges, bridges with three or less girders, and narrow bridges with significant super elevation. Additional restraint, such as XX strong pipe keys, should be considered if stability is questionable after the keys are severely damaged.

7.2.5.4 Hinge Seat Width for Frames Meeting the Requirements of Section 7.1.2

Enough hinge seat width shall be available to accommodate the anticipated thermal movement, prestress shortening, creep, shrinkage, and the relative longitudinal earthquake displacement demand between the two frames calculated by equation 7.6. The seat width normal to the centerline of bearing shall be calculated by equation 7.5 but not less than 24 inches (600 mm).

$$N \geq \begin{cases} \left(\Delta_{p/s} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_{eq} + 4\right) & \text{(in)} \\ \left(\Delta_{p/s} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_{eq} + 100\right) & \text{(mm)} \end{cases} \qquad (7.5)$$

N = Minimum seat width normal to the centerline of bearing

$\Delta_{p/s}$ = Displacement attributed to pre-stress shortening

Δ_{cr+sh} = Displacement attributed to creep and shrinkage

Δ_{temp} = Displacement attributed to thermal expansion and contraction

Δ_{eq} = Relative earthquake displacement demand

$$\Delta_{eq} = \sqrt{\left(\Delta_D^1\right)^2 + \left(\Delta_D^2\right)^2} \qquad (7.6)$$

$\Delta_D^{(i)}$ = The larger earthquake displacement demand for each frame calculated by the global or stand-alone analysis

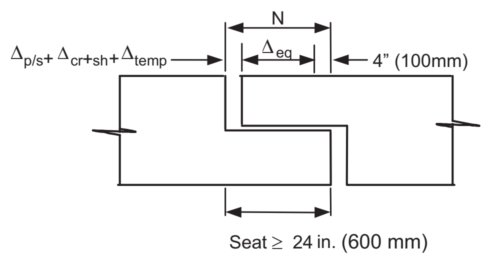

Figure 7.4 Seat Width Requirements

7.2.5.5 Frames Not Meeting the Requirements of Section 7.1.2

Frames that are unbalanced relative to each other have a greater likelihood of responding out-of-phase during earthquakes. Large relative displacements and forces should be anticipated for frames not meeting equation 7.3.

Elastic Analysis, in general, cannot be used to determine the displacement or force demands at the intermediate expansion joints in multi-frame structures. A more sophisticated analysis such as nonlinear dynamic analysis is required that can capture the directivity and time dependency associated with the relative frame displacements. In lieu of nonlinear analysis, the hinge seat can be sized longitudinally and the shear keys isolated transversely to accommodate the absolute sum of the individual frame displacements determined by ESA, EDA, or the initial slope of a "push over" analysis.

Care must be taken to isolate unbalanced frames to insure the seismic demands are not transferred between frames. The following guidelines should be followed when designing and detailing hinges when equation 7.3 is not met.

- Isolate adjacent frames longitudinally by providing a large expansion gap to reduce the likelihood of pounding. Permanent gapping created by prestress shortening, creep, and shrinkage can be considered as part of the isolation between frames.
- Provide enough seat width to reduce the likelihood of unseating. If seat extenders are used they should be isolated transversely to avoid transmitting large lateral shear forces between frames.
- Limit the transverse shear capacity to prevent large lateral forces from being transferred to the stiffer frame. The analytical boundary conditions at the hinge should be either released transversely or able to capture the nonlinear shear friction mechanism expected at the shear key. If the hinges are expected to fail, the column shall be designed to accommodate the displacement demand associated with having the hinge released transversely.

One method for isolating unbalanced frames is to support intermediate expansion joints on closely spaced adjacent bents that can support the superstructure by cantilever beam action. A longitudinal gap is still required to prevent the frames from colliding. Bent supported expansion joints need to be approved on a project-by-project basis, see MTD 20-11.

7.2.6 Hinge Restrainers

A satisfactory method for designing the size and number of restrainers required at expansion joints is not currently available. Adequate seat shall be provided to prevent unseating as a primary requirement. Hinge restrainers are considered secondary members to prevent unseating. The following guidelines shall be followed when designing and detailing hinge restrainers.

- Restrainers design should not be based on the force demands predicted by EDA analysis
- A restrainer unit shall be placed in each alternating cell at all hinges (minimum of two restrainer units at each hinge).
- Restrainers shall be detailed to allow for easy inspection and replacement
- Restrainer layout shall be symmetrical about the centerline of the superstructure
- Restrainer systems shall incorporate an adequate gap for expansion

Yield indicators are required on all cable restrainers, see Standard Detail Sheet XS 12-57.1 for details. See MTD 20-3 for material properties pertaining to high strength rods (ASTM A722 Uncoated High-Strength Steel Bar for Prestressing Concrete) and restrainer cables (ASTM A633 Zinc Coated Steel Structural Wire Rope).

7.2.7 Pipe Seat Extenders

Pipes seat extenders shall be designed for the induced moments under single or double curvature depending on how the pipe is anchored. If the additional support width provided by the pipe seat extender is required to meet equation 7.5 then hinge restrainers are still required. If the pipe seat extenders are provided as a secondary vertical support system above and beyond what is required to satisfy equation 7.5, hinge restrainers are not required. Pipe seat extenders will substantially increase the shear transfer capacity across expansion joints if significant out-of-phase displacements are anticipated. If this is the case, care must be taken to insure stand-alone frame capacity is not adversely affected by the additional demand transmitted between frames through the pipe seat extenders.

7.2.8 Equalizing Bolts

Equalizing bolts are designed for service loads and are considered sacrificial during an earthquake. Equalizing bolts shall be designed so they will not transfer seismic demand between frames or inhibit the performance of the hinge restrainers. Equalizing bolts shall be detailed so they can be easily inspected for damage and/or replaced after an earthquake.

7.3 Bent Caps

7.3.1 Integral Bent Caps

Bent caps are considered integral if they terminate at the outside of the exterior girder and respond monolithically with the girder system during dynamic excitation.

7.3.1.1 Effective Bent Cap Width

The integral cap width considered effective for resisting flexural demands from plastic hinging in the columns shall be determined by equation 7.7. See Figure 7.5.

$$B_{eff} = B_{cap} + (12 \times t) \tag{7.7}$$

t = Thickness of the top or bottom slab

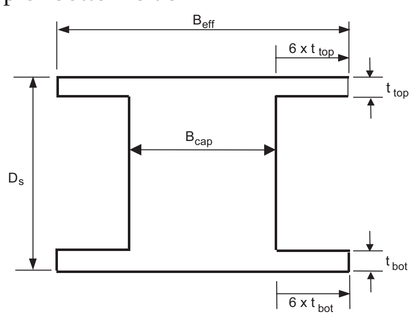

Figure 7.5 Effective Bent Cap Width

7.3.2 Non-Integral Bent Caps

Superstructure members supported on non-integral bent caps shall be simply supported at the bent cap or span continuously with a separation detail such as an elastomeric pad or isolation bearing between the bent cap and the superstructure. Non-integral caps must satisfy all the SDC requirements for frames in the transverse direction.

7.3.2.1 Minimum Bent Cap Seat Width

Drop caps supporting superstructures with expansion joints at the cap shall have sufficient width to prevent unseating. The minimum seat width for non-integral bent caps shall be determined by equation 7.5. Continuity devices such as rigid restrainers or web plates may be used to ensure unseating does not occur but shall not be used in lieu of adequate bent cap width.

7.3.3 Deleted

7.3.4 Bent Cap Depth

Every effort should be made to provide enough cap depth to develop the column longitudinal reinforcement without hooks. See Section 8.2 regarding anchoring column reinforcement into the bent cap.

7.4 Superstructure Joint Design

7.4.1 Joint Performance

Moment resisting connections between the superstructure and the column shall be designed to transmit the maximum forces produced when the column has reached its overstrength capacity M_o^{col} including the effects of overstrength shear V_o^{col}.

7.4.2 Joint Proportioning

All superstructure/column moment resisting joints shall be proportioned so the principal stresses satisfy equations 7.8 and 7.9. See Section 7.4.4.1 for the numerical definition of principal stress.

Principal compression: $p_c \leq 0.25 \times f'_c$ (7.8)

Principal tension: $p_t \leq 12 \times \sqrt{f'_c}$ (psi) $p_t \leq 1.0 \times \sqrt{f'_c}$ (MPa) (7.9)

7.4.2.1 Minimum Bent Cap Width

The minimum bent cap width required for adequate joint shear transfer is specified in equation 7.10. Larger cap widths may be required to develop the compression strut outside the joint for large diameter columns.

$$B_{cap} = D_c + 2 \quad \text{(ft)} \qquad\qquad B_{cap} = D_c + 600 \quad \text{(mm)} \tag{7.10}$$

7.4.3 Joint Description

The following types of joints are considered T joints for joint shear analysis:

- Integral interior joints of multi-column bents in the transverse direction.
- All column/superstructure joints in the longitudinal direction.
- Exterior column joints for box girder superstructures if the cap beam extends beyond the joint far enough to develop the longitudinal cap reinforcement.[8]

7.4.4 T Joint Shear Design

7.4.4.1 Principal Stress Definition

The principal tension and compression stresses in a joint are defined as follows:

$$p_t = \frac{(f_h + f_v)}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} \tag{7.11}$$ [9]

$$p_c = \frac{(f_h + f_v)}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jv}^2} \tag{7.12}$$

$$v_{jv} = {}^{T_c}\!/_{A_{jv}} \tag{7.13}$$

$$A_{jv} = l_{ac} \times B_{cap} \tag{7.14}$$ [10]

$$f_v = \frac{P_c}{A_{jh}} \tag{7.15}$$

$$A_{jh} = (D_c + D_s) \times B_{cap} \tag{7.16}$$

$$f_h = \frac{P_b}{B_{cap} \times D_s} \tag{7.17}$$

---

8 All other exterior joints are considered knee joints in the transverse direction. Knee joints are nonstandard elements, design criteria shall be developed on a project specific basis.

9 A negative result from equation 7.11 signifies the joint has nominal principal tensile stresses.

10 Equation 7.14 defines the effective joint area in terms of the bent cap width regardless of the direction of bending. This lone simplified definition of A_{jv} may conservatively underestimate the effective joint area for columns with large cross section aspect ratios in longitudinal bending.

Where:

A_{jh}	=	The effective horizontal joint area
A_{jv}	=	The effective vertical joint area
B_{cap}	=	Bent cap width
D_c	=	Cross–sectional dimension of column in the direction of bending
D_s	=	Depth of superstructure at the bent cap
l_{ac}	=	Length of column reinforcement embedded into the bent cap
P_c	=	The column axial force including the effects of overturning
P_b	=	The beam axial force at the center of the joint including prestressing
T_c	=	The column tensile force defined as M_o^{col}/h, where h is the distance from c.g. of tensile force to c.g. of compressive force on the section, or alternatively T_c may be obtained from the moment-curvature analysis of the cross section.

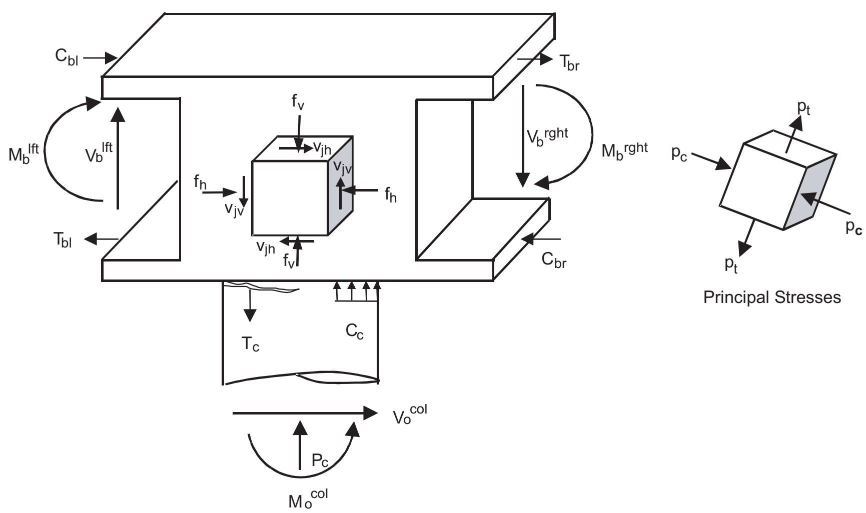

Figure 7.6 Joint Shear Stresses in T Joints

Note: Unless the prestressing is specifically designed to provide horizontal joint compression, f_h can typically be ignored without significantly effecting the principal stress calculation.

7.4.4.2 Minimum Joint Shear Reinforcement

If the principal tension stress p_t does not exceed $3.5 \times \sqrt{f'_c}$ psi ($0.29 \times \sqrt{f'_c}$ MPa) the minimum joint shear reinforcement, as specified in equation 7.18, shall be provided. This joint shear reinforcement may be provided in the form of column transverse steel continued into the bent cap. No additional joint reinforcement is required. The volumetric ratio of transverse column reinforcement ρ_s continued into the cap shall not be less than the value specified by equation 7.18.

$$\rho_{s,\,min} = \frac{3.5 \times \sqrt{f'_c}}{f_{yh}} \quad \text{(psi)} \qquad \frac{0.29 \times \sqrt{f'_c}}{f_{yh}} \quad \text{(MPa)} \tag{7.18}$$

The reinforcement shall be in the form of spirals, hoops, or intersecting spirals or hoops.

If the principal tension stress p_t exceeds $3.5\times\sqrt{f'_c}$ psi ($0.29\times\sqrt{f'_c}$ MPa) the joint shear reinforcement specified in Section 7.4.4.3 is required.

7.4.4.3 Joint Shear Reinforcement

A) Vertical Stirrups:

$$A_s^{jv} = 0.2\times A_{st} \tag{7.19}$$

A_{st} = Total area of column reinforcement anchored in the joint

Vertical stirrups or ties shall be placed transversely within a distance D_c extending from either side of the column centerline. The vertical stirrup area, A_{jv} is required on each side of the column or pier wall, see Figures 7.7, 7.8, and 7.10. The stirrups provided in the overlapping areas shown in Figure 7.7 shall count towards meeting the requirements of both areas creating the overlap. These stirrups can be used to meet other requirements documented elsewhere including the shear in the bent cap.

B) Horizontal Stirrups:

Horizontal stirrups or ties shall be placed transversely around the vertical stirrups or ties in two or more intermediate layers spaced vertically at not more than 18 inches (450mm). This horizontal reinforcement A_s^{jh} shall be placed within a distance D_c extending from either side of the column centerline, see Figure 7.9.

$$A_s^{jh} = 0.1\times A_{st} \tag{7.20}$$

C) Horizontal Side Reinforcement:

The total longitudinal side face reinforcement in the bent cap shall be at least equal to the greater of the areas specified in equation 7.21 and shall be placed near the side faces of the bent cap with a maximum spacing of 12 inches (300mm), see Figure 7.8. Any side reinforcement placed to meet other requirements shall count towards meeting the requirement in this section.

$$A_s^{sf} \ge \begin{cases} 0.1\times A_{cap}^{top} \\ \text{or} \\ 0.1\times A_{cap}^{bot} \end{cases} \qquad A_{cap} = \text{Area of bent cap top or bottom flexural steel} \tag{7.21}$$

D) J-Dowels

For bents skewed greater than 20°, J-dowels hooked around the longitudinal top deck steel extending alternatively 24 inches (600 mm) and 30 inches (750 mm) into the bent cap are required. The J-dowel reinforcement shall be equal or greater than the area specified in equation 7.22.

$$A_s^{\text{j-bar}} = 0.08\times A_{st} \tag{7.22}$$

The J-dowels shall be placed within a rectangular region defined by the width of the bent cap and the distance D_c on either side of the centerline of the column, see Figure 7.10.

E) Transverse Reinforcement

Transverse reinforcement in the joint region shall consist of hoops with a minimum reinforcement ratio specified by equation 7.23. The column confinement reinforcement extended into the bent cap may be used to meet this requirement.

$$\rho_s = 0.4 \times \frac{A_{st}}{l_{ac}^2} \qquad \text{(in, mm)} \qquad (7.23)$$

For interlocking cores ρ_s shall be based on area of reinforcement (A_{st}) of each core.
All vertical column bars shall be extended as close as possible to the top bent cap reinforcement.

F) Main Column Reinforcement

The main column reinforcement shall extend into the cap as deep as possible to fully develop the compression strut mechanism in the joint.

7.4.5 Knee Joints

Knee joints differ from T joints because the joint response varies with the direction of the moment (opening or closing) applied to the joint. Knee joints require special reinforcing details that are considered non-standard and shall be included in the project specific seismic design criteria.

It may be desirable to pin the top of the column to avoid knee joint requirements. This eliminates the joint shear transfer through the joint and limits the torsion demand transferred to the cap beam. However, the benefits of a pinned exterior joint should be weighed against increased foundation demands and the effect on the frame's overall performance.

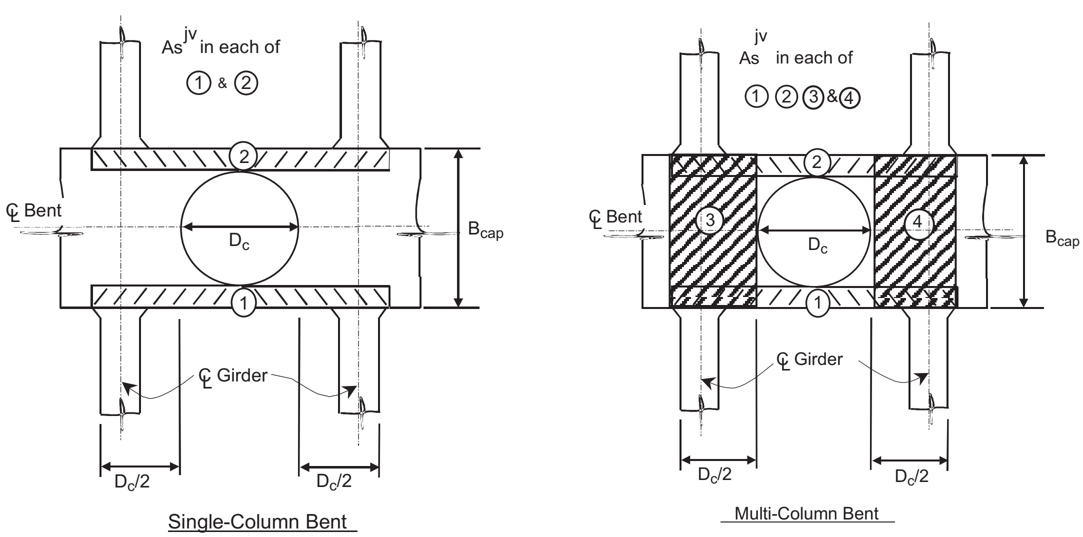

Figure 7.7 Location of Vertical Joint Reinforcement (Plan View of Bridge)

Bent Cap Details, Section at Column for Bridges with 0 to 20-Degree Skew.
(Detail Applies to Sections Within 2 x Diameter of Column, Centered About CL of Column).
(Detail Applies to T-Beam and Box Girder Bridges Where Deck Reinforcement is Placed Parallel to Cap).

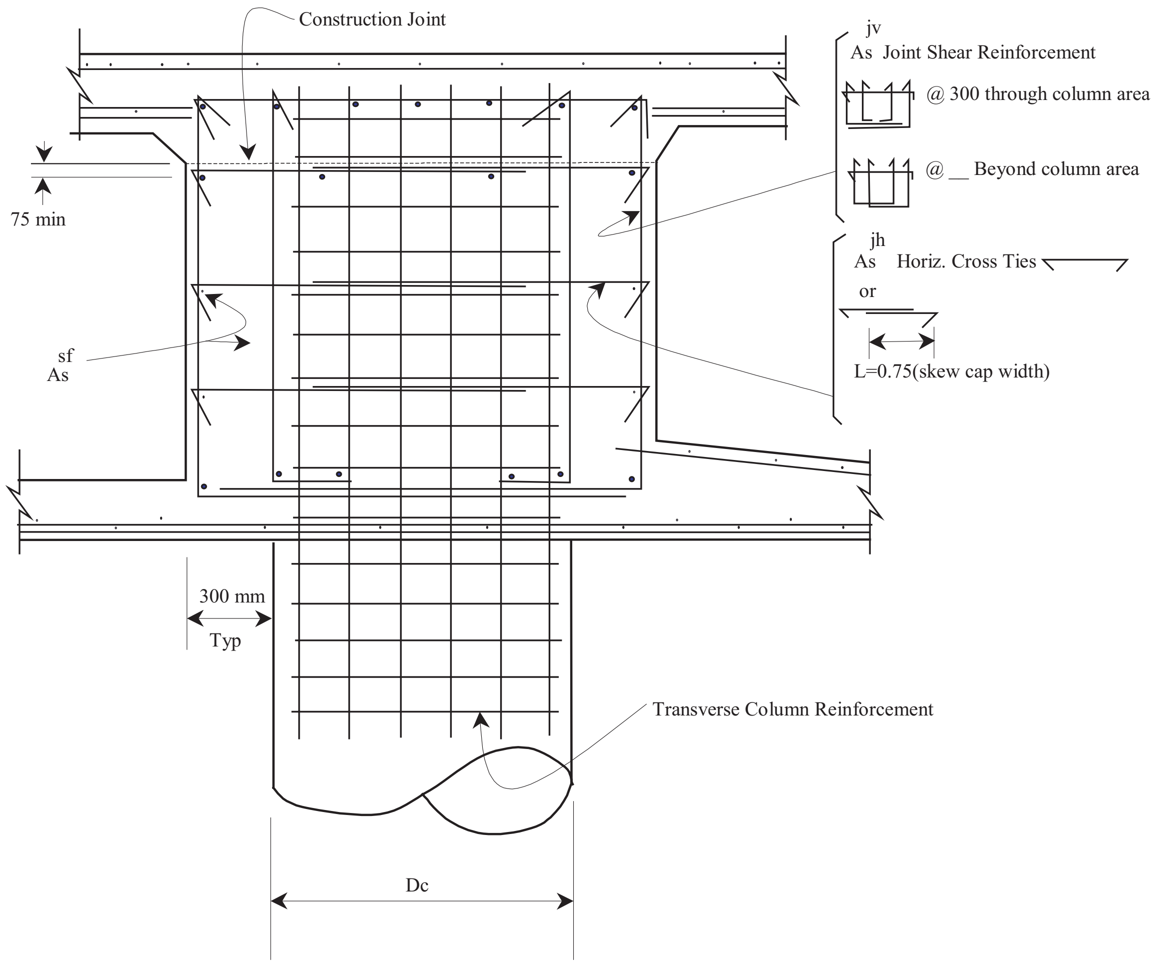

Figure 7.8 Joint Shear Reinforcement Details[11]

[11] Figures 7.8, 7.9, and 7.10 illustrate the general location for joint shear reinforcement in the bent cap.

Bent Cap Elevation.
Horizontal Cross Tie and J-bar Placing Pattern.

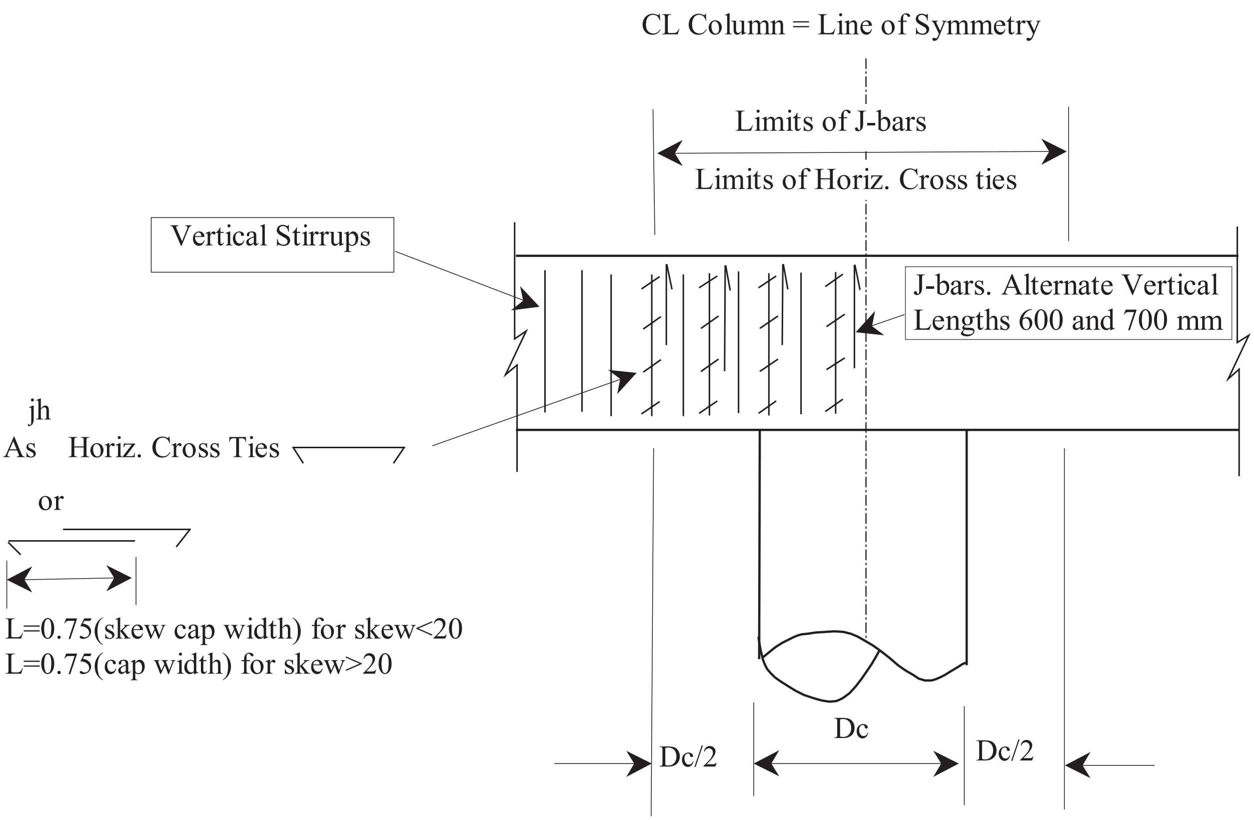

Figure 7.9 Location of Horizontal Joint Shear Steel[12]

[12] Figures 7.8, 7.9, and 7.10 illustrate the general location for joint shear reinforcement in the bent cap.

Bent Cap Details, Section at Column for Bridges with Skew Larger than 20 Degrees.
(Detail Applies to Sections Within 2 x Diameter of Column, Centered About CL of Column).
(Detail Applies to T-Beam and Box Girder Bridges Where Deck Reinforcement is Placed Normal or Radial to CL Bridge).

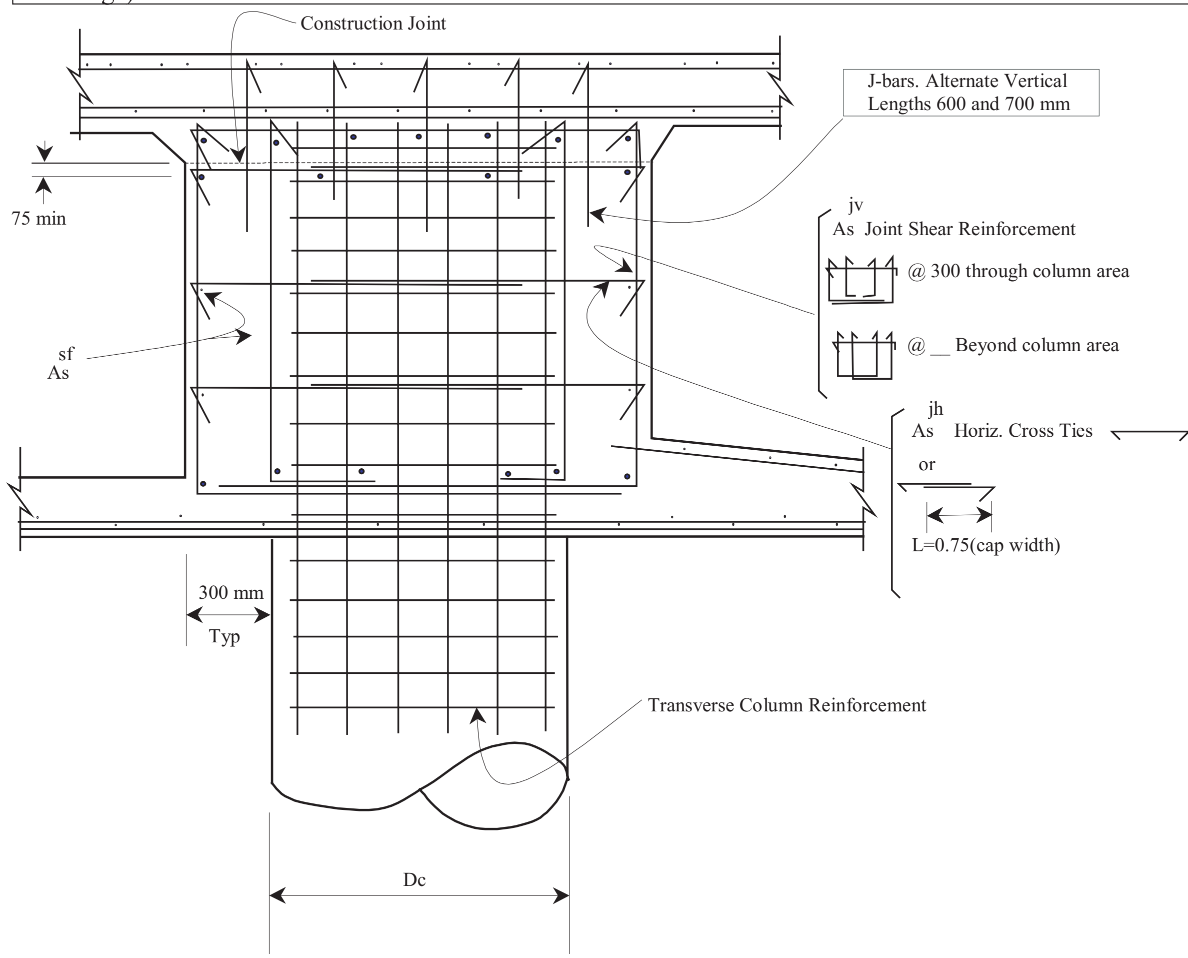

Figure 7.10 Additional Joint Shear Steel for Skewed Bridges[13]

[13] Figures 7.8, 7.9, and 7.10 illustrate the general location for joint shear reinforcement in the bent cap.

7.5 Bearings

For Ordinary Standard bridges bearings are considered sacrificial elements. Typically bearings are designed and detailed for service loads. However, bearings shall be checked to insure their capacity and mode of failure are consistent with the assumptions made in the seismic analysis. The designer should consider detailing bearings so they can be easily inspected for damage and replaced or repaired after an earthquake.

7.5.1 Elastomeric Bearings

The lateral shear capacity of elastomeric bearing pads is controlled by either the dynamic friction capacity between the pad and the bearing seat or the shear strain capacity of the pad. Test results have demonstrated the dynamic coefficient of friction between concrete and neoprene is 0.40 and between neoprene and steel is 0.35. The maximum shear strain resisted by elastomeric pads prior to failure is estimated at $\pm 150\%$.

7.5.2 Sliding Bearings

PTFE spherical bearings and PTFE elastomeric bearings utilize low friction PTFE sheet resin. Typical friction coefficients for these bearings vary between 0.04 to 0.08. The friction coefficient is dependent on contact pressure, temperature, sliding speed, and the number of sliding cycles. Friction values may be as much as 5 to 10 times higher at sliding speeds anticipated under seismic loads compared to the coefficients under thermal expansion.

A common mode of failure for sliding bearings under moderate earthquakes occurs when the PTFE surface slides beyond the limits of the sole plate often damaging the PTFE surface. The sole plate should be extended a reasonable amount to eliminate this mode of failure whenever possible.

7.6 Columns & Pier Walls

7.6.1 Column Dimensions

Every effort shall be made to limit the column cross sectional dimensions to the depth of the superstructure. This requirement may be difficult to meet on columns with high L/D ratios. If the column dimensions exceed the depth of the bent cap it may be difficult to meet the joint shear requirements in Section 7.4.2, the superstructure capacity requirements in Section 4.3.2.1, and the ductility requirements in Section 3.1.4.1.

The relationship between column cross section and bent cap depth specified in equation 7.24 is a guideline based on observation. Maintaining this ratio should produce reasonably well proportioned structures.

$$0.7 < \frac{D_c}{D_s} < 1.0 \qquad (7.24)$$

7.6.2 Analytical Plastic Hinge Length

The analytical plastic hinge length is the equivalent length of column over which the plastic curvature is assumed constant for estimating plastic rotation.

7.6.2 (a) Columns & Type II Shafts:

$$L_p = \begin{cases} 0.08L + 0.15 f_{ye} d_{bl} \geq 0.3 f_{ye} d_{bl} & \text{(in, ksi)} \\ 0.08L + 0.022 f_{ye} d_{bl} \geq 0.044 f_{ye} d_{bl} & \text{(mm, MPa)} \end{cases} \tag{7.25}$$

7.6.2 (b) Horizontally Isolated Flared Columns

$$L_p = \begin{cases} G + 0.3 f_{ye} d_{bl} & \text{(in, ksi)} \\ G + 0.044 f_{ye} d_{bl} & \text{(mm, MPa)} \end{cases} \tag{7.26}$$

G = The gap between the isolated flare and the soffit of the bent cap

7.6.2 (c) Non-cased Type I Pile Shafts:

$$L_p = D^* + 0.08H' \tag{7.27}$$

D^* = Diameter for circular shafts or the least cross section dimension for oblong shafts.

H' = Length of pile shaft/column from point of maximum moment to point of contra-flexure above ground considering the base of plastic hinge at the point of maximum moment.

7.6.3 Plastic Hinge Region

The plastic hinge region, L_{pr} defines the portion of the column, pier, or shaft that requires enhanced lateral confinement. L_{pr} is defined by the larger of:

- 1.5 times the cross sectional dimension in the direction of bending
- The region of column where the moment exceeds 75% of the maximum plastic moment, M_p^{col}
- 0.25(Length of column from the point of maximum moment to the point of contra-flexure)

7.6.4 Multi-Column Bents

The effects of axial load redistribution due to overturning forces shall be considered when calculating the plastic moment capacity for multi-column bents in the transverse direction.

7.6.5 Column Flares

7.6.5.1 Horizontally Isolated Column Flares

The preferred method for detailing flares is to horizontally isolate the top of flared sections from the soffit of the cap beam. Isolating the flare allows the flexural hinge to form at the top of the column, minimizing the seismic shear demand on the column. The added mass and stiffness of the isolated flare typically can be ignored in the dynamic analysis.

A horizontal gap isolating the flare from the cap beam shall extend over the entire cross section of the flare excluding a core region equivalent to the prismatic column cross section. The gap shall be large enough so that it will not close during a seismic event. The gap thickness, G shall be based on the estimated ductility demand and corresponding plastic hinge rotation capacity. The minimum gap thickness shall be 2 inches (50 mm). See Section 7.6.2 for the appropriate plastic hinge length of horizontally isolated flares.

If the plastic hinge rotation based on the plastic hinge length specified Section 7.6.2 (b) provides insufficient column displacement capacity, the designer may elect to add vertical flare isolation. When vertical flare isolation is used, the analytical plastic hinge length shall be taken as the lesser of L_p calculated using Equations 7.25 and 7.26 where G is the length from the bent cap soffit to the bottom of the vertical flare isolation region.[14]

7.6.5.2 Lightly Reinforced Column Flares

Column flares that are integrally connected to the bent cap soffit should be avoided whenever possible. Lightly reinforced integral flares shall only be used when required for service load design or aesthetic considerations and the peak rock acceleration is less than 0.5*g*. The flare geometry shall be kept as slender as possible. Test results have shown that slender lightly reinforced flares perform adequately after cracking has developed in the flare concrete essentially separating the flare from the confined column core. However, integral flares require higher shear forces and moments to form the plastic hinge at the top of column compared to isolated flares. The column section at the base of the flare must have adequate capacity to insure the plastic hinge will form at the top of column. The higher plastic hinging forces must be considered in the design of the column, superstructure and footing.

7.6.5.3 Flare Reinforcement

Column flares shall be nominally reinforced outside the confined column core to prevent the flare concrete from completely separating from the column at high ductility levels.

7.6.6 Pier Walls

Pier walls shall be designed to perform in a ductile manner longitudinally (about the weak axis), and to remain essentially elastic in the transverse direction (about the strong axis). The large difference in stiffness between the strong and weak axis of pier walls leads to complex foundation behavior, see Section 7.7.

7.6.7 Column Key Design

Column shear keys shall be designed for the axial and shear forces associated with the column's overstrength moment M_o^{col} including the effects of overturning. The key reinforcement shall be located as close to the center of the column as possible to minimize developing a force couple within the key reinforcement. Steel pipe sections may be used in lieu of reinforcing steel to relieve congestion and reduce the moment generated within the key. Any appreciable moment generated by the key steel should be considered in the footing design.

[14] The horizontal flare isolation detail is easier to construct than a combined horizontal and vertical isolation detail and is preferred wherever possible. Laboratory testing is scheduled to validate the plastic hinge length specified in equation 7.26.

7.7 Foundations

7.7.1 Footing Design

7.7.1.1 Pile Foundations in Competent Soil

The lateral, vertical, and rotational capacity of the foundation shall exceed the respective demands. The size and number of piles and the pile group layout shall be designed to resist service level moments, shears, and axial loads and the moment demand induced by the column plastic hinging mechanism. Equations 7.28 and 7.29 define lateral shear and moment equilibrium in the foundation when the column reaches its overstrength capacity, see Figure 7.11.

$$V_o^{col} - \sum V_{(i)}^{pile} - R_s = 0 \tag{7.28}$$

$$M_o^{col} + V_o^{col} \times D_{ftg} + \sum M_{(i)}^{pile} - R_s \times (D_{ftg} - D_{R_s}) - \sum (C_{(i)}^{pile} \times c_{(i)}) - \sum (T_{(i)}^{pile} \times c_{(i)}) = 0 \tag{7.29}$$

$c_{(i)}$ = Distance from pile (i) to the center of gravity of the pile group in the X or Y direction

$C_{(i)}^{pile}$ = Axial compression demand on pile (i)

D_{ftg} = Depth of footing

D_{R_s} = Depth of resultant soil resistance measured from the top of footing

$M_{(i)}^{pile}$ = The moment demand generated in pile (i), $M_{(i)}^{pile} = 0$ if the piles are pinned to the footing

R_s = Estimated resultant soil resistance on the end of the footing

$T_{(i)}^{pile}$ = Axial tension demand on pile (i)

$V_{(i)}^{pile}$ = Lateral shear resistance provided by pile (i)

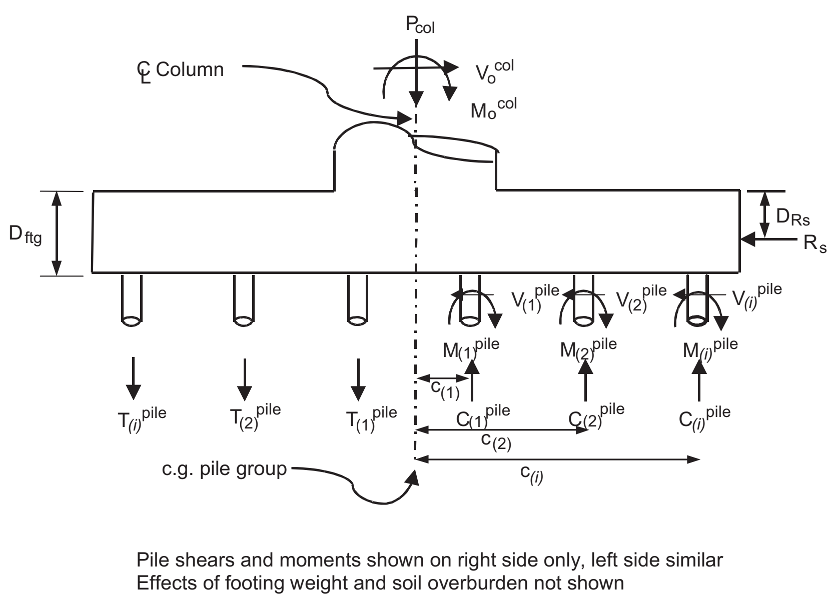

Figure 7.11 Footing Force Equilibrium

The design of pile foundations in competent soil can be greatly simplified if we rely on inherent capacity that is not directly incorporated in the foundation assessment. For example, typically pile axial resistance exceeds the designed nominal resistance and axial load redistributes to adjacent piles when an individual pile's geotechnical capacity is exceeded.

The simplified foundation model illustrated in Figure 7.12 is based on the following assumptions. A more sophisticated analysis may be warranted if project specific parameters invalidate any of these assumptions:

- The passive resistance of the soil along the leading edge of the footing and upper 4 to 8 pile diameters combined with the friction along the sides and bottom of the pile cap is sufficient to resist the column overstrength shear V_o^{col}.
- The pile cap is infinitely rigid, its width is entirely effective, and the pile loads can be calculated from the static equations of equilibrium.
- The pile group's nominal moment resistance is limited to the capacity available when any individual pile reaches its nominal axial resistance.
- Group effects for pile footings surrounded by competent soil and a minimum of three diameters center-to-center pile spacing are relatively small and can be ignored.
- Piles designed with a pinned connection to the pile cap will not transfer significant moment to the pile cap.
- Pile groups designed with the simplified foundation model can be sized to resist the plastic moment of the column M_p in lieu of M_o.

Equation 7.30 defines the axial demand on an individual pile when the column reaches its plastic hinging capacity based on force equilibrium in conjunction with the previously stated assumptions. A similar model can be used to analyze and design spread footing foundations that are surrounded by competent soil.

$$\left.\begin{array}{l} C_{(i)}^{pile} \\ \\ T_{(i)}^{pile} \end{array}\right\} = \frac{P_c}{N} \pm \frac{M_{p(y)}^{col} \times c_{x(i)}}{I_{p.g.(y)}} \pm \frac{M_{p(x)}^{col} \times c_{y(i)}}{I_{p.g.(x)}} \tag{7.30}$$

$$I_{p.g.(y)} = \sum n \times c_{y(i)}^2 \qquad I_{p.g.(x)} = \sum n \times c_{x(i)}^2 \tag{7.31}$$

Where:

$I_{p.g.}$	=	Moment of inertia of the pile group defined by equation 7.31
$M_{p(y),(x)}^{col}$	=	The component of the column plastic moment capacity about the X or Y axis
N_p	=	Total number of piles in the pile group
n	=	The total number of piles at distance $c_{(i)}$ from the centroid of the pile group
P_c	=	The total axial load on the pile group including column axial load (dead load+EQ load), footing weight, and overburden soil weight

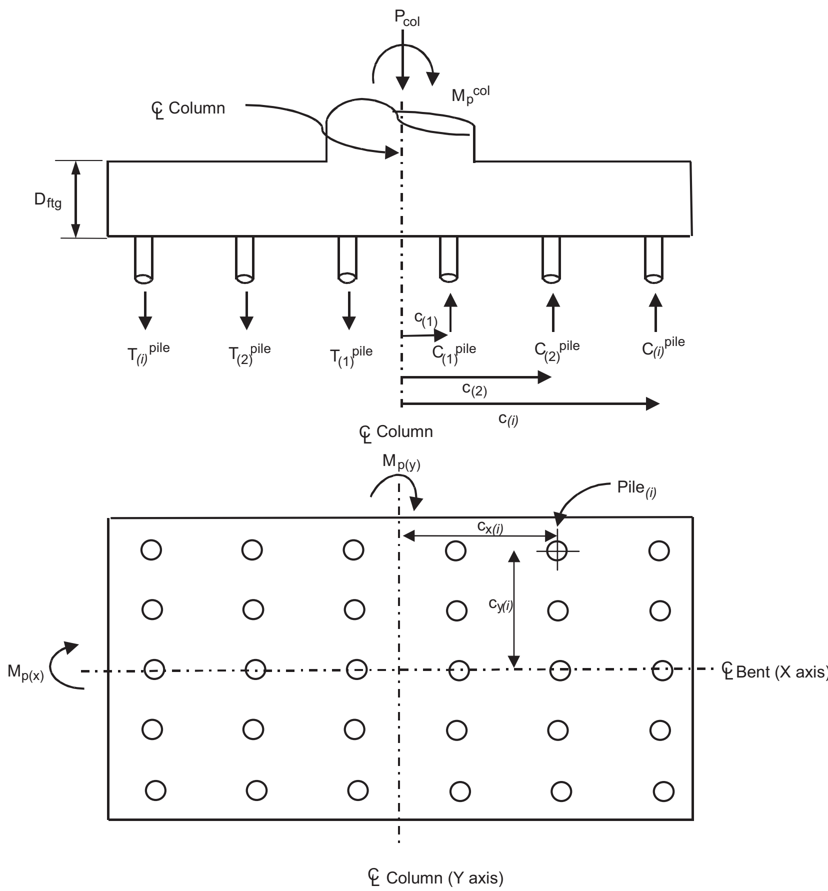

Figure 7.12 Simplified Pile Model for Foundations in Competent Soil

7.7.1.2 Pile Foundations in Marginal Soil

7.7.1.2.1 Lateral Design

In marginal soils the pile cap may not dominate the lateral stiffness of the foundation, as is expected in competent soil, possibly leading to significant lateral displacements. The designer shall verify that the lateral capacity of the foundation exceeds the lateral demand transmitted by the column, including the pile's capability of maintaining axial load capacity at the expected lateral displacement.

The designer should select the most cost effective strategy for increasing the lateral resistance of the foundation when required. The following methods are commonly used to increase lateral foundation capacity.

- Deepen the footing/pile cap to increase passive resistance
- Increase the amount of fixity at the pile/footing connection and strengthen the upper portion of the pile
- Use a more ductile pile type that can develop soil resistance at larger pile deflections
- Add additional piles

7.7.1.2.2 *Lateral Capacity of Fixed Head Piles*

The lateral capacity assessment of fixed head piles requires a project specific design which considers the effects of shear, moment, axial load, stiffness, soil capacity, and stability.

7.7.1.2.3 *Passive Earth Resistance for Pile Caps in Marginal Soil*

Assessing the passive resistance of the soil surrounding pile caps under dynamic loading is complex. The designer may conservatively elect to ignore the soil's contribution in resisting lateral loads. In this situation, the piles must be capable of resisting the entire lateral demand without exceeding the force or deformation capacity of the piles.

Alternatively, contact the Project Geologist/Geotechnical Engineer to obtain force deformation relationships for the soil that will be mobilized against the footing. The designer should bear in mind that significant displacement may be associated with the soil's ultimate passive resistance.

7.7.1.3 *Rigid Footing Response*

The length to thickness ratio along the principal axes of the footing must satisfy equation 7.32 if rigid footing behavior and the associated linear distribution of pile forces and deflections is assumed.

$$L_{ftg}/D_{ftg} \le 2.5 \tag{7.32}$$

L_{ftg} = The cantilever length of the pile cap measured from the face of the column to the edge of the footing.

7.7.1.4 *Footing Joint Shear*

All footing/column moment resisting joints shall be proportioned so the principal stresses meet the following criteria:

Principal compression: $$p_c \le 0.25 \times f'_c \tag{7.33}$$

Principal tension: $$p_t \le \begin{cases} 12 \times \sqrt{f'_c} & \text{(psi)} \\ 1.0 \times \sqrt{f'_c} & \text{(MPa)} \end{cases} \tag{7.34}$$

Where:

$$p_t = \frac{f_v}{2} - \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \tag{7.35}$$

$$p_c = \frac{f_v}{2} + \sqrt{\left(\frac{f_v}{2}\right)^2 + v_{jv}^2} \tag{7.36}$$

$$v_{jv} = \frac{T_{jv}}{B_{eff}^{ftg} \times D_{ftg}} \tag{7.37}$$

$$T_{jv} = T_c - \sum T_{(i)}^{pile} \tag{7.38}$$

T_c = Column tensile force associated with M_o^{col}

$\sum T_{(i)}^{pile}$ = Summation of the hold down force in the tension piles.

$$B_{eff}^{ftg} = \begin{cases} \sqrt{2} \times D_c & \text{Circular Column} \\ B_c + D_c & \text{Rectangular Column} \end{cases} \tag{7.39}$$

$$f_v = \frac{P_c}{A_{jh}^{ftg}} \tag{7.40}$$

$$A_{jh}^{ftg} = \begin{cases} \left(D_c + D_{ftg}\right)^2 & \text{Circular Column} \\ \left(D_c + \dfrac{D_{ftg}}{2}\right) \times \left(B_c + \dfrac{D_{ftg}}{2}\right) & \text{Rectangular Column} \end{cases} \tag{7.41}$$

Where:

A_{jh}^{ftg} is the effective horizontal area at mid-depth of the footing, assuming a 45° spread away from the boundary of the column in all directions, see Figure 7.13.

7.7.1.5 Effective Footing Width for Flexure

If the footing is proportioned according to Sections 7.7.1.3 and 7.7.1.4 the entire width of the footing can be considered effective in resisting the column overstrength flexure and the associated shear.

7.7.1.6 Effects of Large Capacity Piles on Footing Design

The designer shall insure the footing has sufficient strength to resist localized pile punching failure for piles exceeding nominal resistance of 400 kips (1800kN). In addition, a sufficient amount of the flexure reinforcement in the top and bottom mat must be developed beyond the exterior piles to insure tensile capacity is available to resist the horizontal component of the shear-resisting mechanism for the exterior piles.

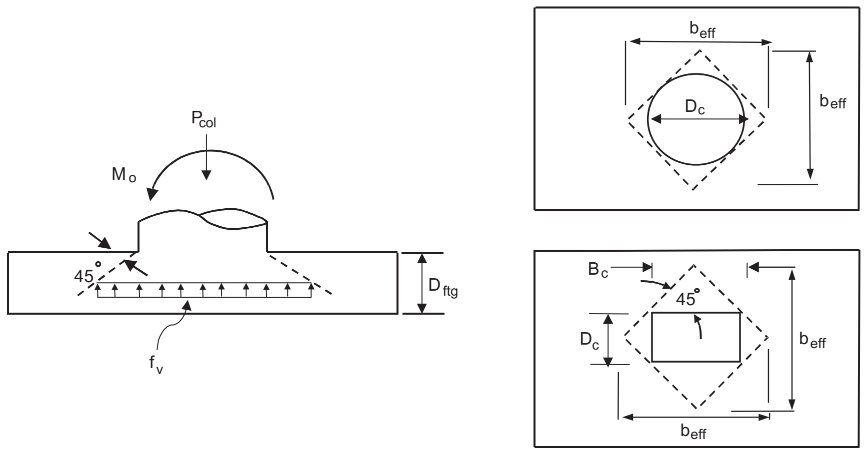

Figure 7.13 Effective Joint Width for Footing Joint Stress Calculation

7.7.2 Pier Wall Pile Foundations

Typically, it is not economical to design pier wall pile foundations to resist the transverse seismic shear. Essentially elastic response of the wall in the strong direction will induce large foundation demands that may cause inelastic response in the foundation. If this occurs, piles will incur some damage from transverse demands, most likely near the pile head/pile cap connection. Methods for reducing the inelastic damage in pier wall pile foundations include:

- Utilizing ductile pile head details
- Pinning the pier wall-footing connection in the weak direction to reduce the weak axis demand on the piles that may be damaged by transverse demands
- Pinning the pier wall-soffit connection, thereby limiting the demands imparted to the substructure
- Use a ductile system in lieu of the traditional pier wall. For example, columns or pile extensions with isolated shear walls

The method selected to account for or mitigate inelastic behavior in the pier wall foundations shall be discussed at the Type Selection Meeting.

7.7.2.1 Pier Wall Spread Footing Foundations

If sliding of the pier wall foundation is anticipated, the capacity of the pier wall and foundation must be designed for 130% of a realistic estimate of the sliding resistance at the bottom of the footing.

7.7.3 Pile Shafts

7.7.3.1 Shear Demand on Type I Pile Shafts

Overestimating the equivalent cantilever length of pile shafts will under estimate the shear load corresponding to the plastic capacity of the shaft. The seismic shear force for Type I pile shafts shall be taken as the larger of either the shear reported from the soil/pile interaction analysis when the in-ground plastic hinges forms, or the shear calculated by dividing the overstrength moment capacity of the pile shaft by H_s. Where H_s is defined as the smaller length specified by equation 7.42.

$$H_s \leq \begin{cases} H' + (2 \times D_c) \\ \text{Length of the column/shaft from the point of maximum moment} \\ \text{in the shaft to the point contraflexure in the column} \end{cases} \tag{7.42}$$

7.7.3.2 Flexure Demand/Capacity Requirements for Type II Pile Shafts

The distribution of moment along a pile shaft is dependent upon the geotechnical properties of the surrounding soil and the stiffness of the shaft. To ensure the formation of plastic hinges in columns and to minimize the damage to type II shafts a factor of safety of 1.25 shall be used in the design of Type II shafts. This factor also accommodates the uncertainty associated with estimates on soil properties and stiffness. The expected nominal moment capacity M_{ne}^{typeII}, at any location along the shaft, must be at least 1.25 times the moment demand generated by the overstrength moment applied at the base of the column. Increasing the pile shaft's capacity to meet the overstrength requirement will affect the moment demand in the shaft. This needs to be considered and may require iteration to achieve the specified overstrength.

7.7.3.3 Pile Shaft Diameter

Pile shaft construction practice often requires the use of temporary casing (straight or telescoping) especially in the upper 20 feet (6 m). Pile shafts diameters are commonly 6 inches (150 mm) larger than specified when straight casing is used, and 1 foot (300 mm) larger for each piece of telescoping casing. The effect of oversized shafts on the foundation's performance should be considered.

7.7.3.4 Minimum Pile Shaft Length

Pile shafts must have sufficient length to ensure stable load-deflection characteristics.

7.7.3.5 Enlarged Pile Shafts

Type II shafts typically are enlarged relative to the column diameter to contain the inelastic action to the column. Enlarged shafts shall be at least 18 inches (450 mm) larger than the column diameter and the reinforcement shall satisfy the clearance requirements for CIP piling specified in Bridge Design Details 13-22.

7.7.4 Pile Extensions

Pile extensions must perform in a ductile manner and meet the ductility requirements of column elements specified in Section 4.1.

7.8 Abutments

7.8.1 Longitudinal Abutment Response

The linear elastic demand model shall include an effective abutment stiffness, K_{eff} that accounts for expansion gaps, and incorporates a realistic value for the embankment fill response. The abutment embankment fill stiffness is nonlinear and is dependent upon on the material properties of the abutment backfill. Based on passive earth pressure tests and the force deflection results from large-scale abutment testing at UC Davis, the initial embankment fill stiffness is $K_i \approx 20\,\frac{\text{kip/in}}{\text{ft}}$ $(11.5\,\frac{kN/mm}{m})$. The initial stiffness[15] shall be adjusted proportional to the backwall/diaphragm height, as documented in Equation 7.43.

$$K_{abut} = \begin{cases} K_i \times w \times \left(\dfrac{h}{5.5}\right) & \text{U.S. units} \\ K_i \times w \times \left(\dfrac{h}{1.7}\right) & \text{S.I. units} \end{cases} \tag{7.43}$$

Where, w is the width of the backwall or the diaphragm for seat and diaphragm abutments, respectively.

The passive pressure resisting the movement at the abutment increases linearly with the displacement, as shown in Figure 7.14A. The maximum passive pressure of 5.0 ksf (239 kPa), presented in Equation 7.44 is based on the ultimate static force developed in the full scale abutment testing conducted at UC Davis [Maroney, 1995]. The height proportionality factor, $h/5.5\ ft\ \left(h/1.7\ m\right)$ is based on the height of the UC Davis abutment specimen 5.5 ft (1.7 m).

$$P_{bw}\ or\ P_{dia} = \begin{cases} A_e \times 5.0\ ksf \times \left(\dfrac{h_{bw}\ or\ h_{dia}}{5.5}\right) & (ft, kip) \\ A_e \times 239\ kPa \times \left(\dfrac{h_{bw}\ or\ h_{dia}}{1.7}\right) & (m, kN) \end{cases} \tag{7.44}$$

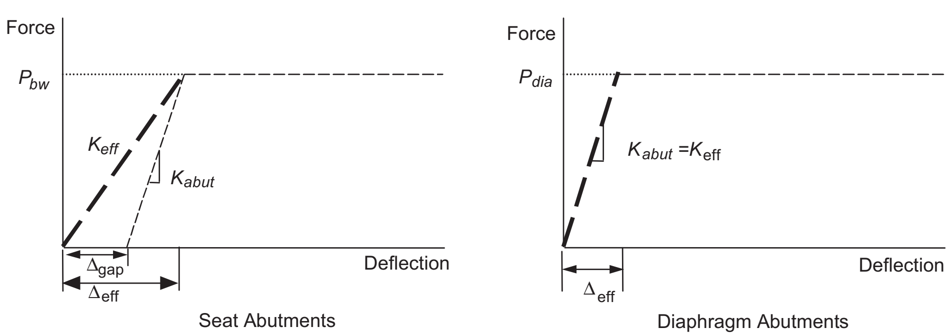

Figure 7.14A Effective Abutment Stiffness

The effective abutment area for calculating the ultimate longitudinal force capacity of an abutment is presented in Equation 7.45.

[15] This proportionality may be revised in future as more data becomes available.

For seat abutments the backwall is typically designed to break off in order to protect the foundation from inelastic action. The area considered effective for mobilizing the backfill longitudinally is equal to the area of the backwall.

For diaphragm abutments the entire diaphragm, above and below the soffit, is typically designed to engage the backfill immediately when the bridge is displaced longitudinally. Therefore, the effective abutment area is equal to the entire area of the diaphragm. If the diaphragm has not been designed to resist the passive earth pressure exerted by the abutment backfill, the effective abutment area is limited to the portion of the diaphragm above the soffit of the girders.

$$A_e = \begin{cases} h_{bw} \times w_{bw} & \text{Seat Abutments} \\ h_{dia} \times w_{dia} & \text{Diaphragm Abutments} \end{cases} \tag{7.45}$$

$h_{dia} = h_{dia}^{*}$ = Effective height if the diaphragm is not designed for full soil pressure (see Figure 7.14B).

$h_{dia} = h_{dia}^{**}$ = Effective height if the diaphragm is designed for full soil pressure (see Figure 7.14B).

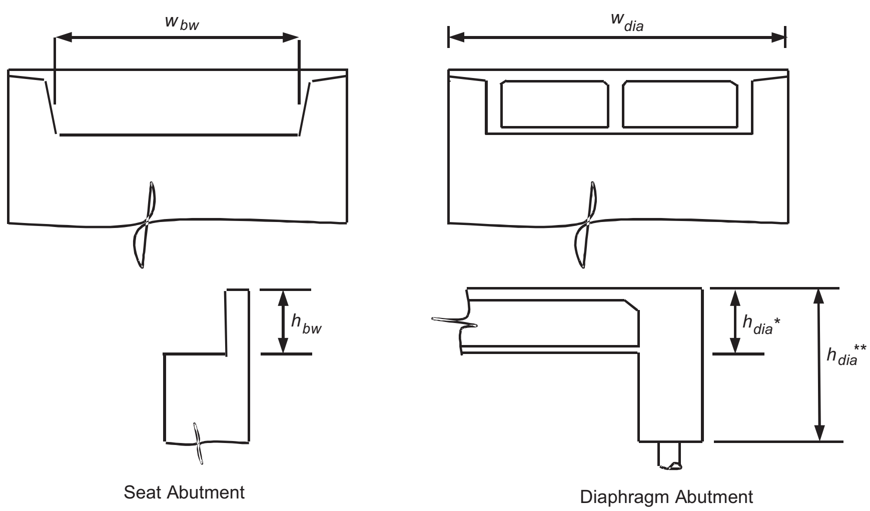

Figure 7.14B Effective Abutment Area

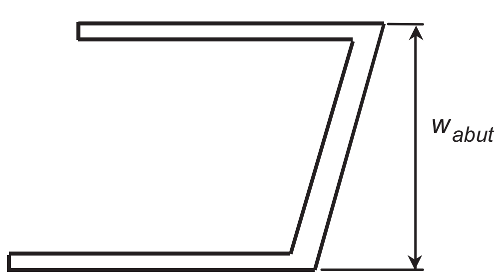

Figure 7.14C Effective Abutment Width for Skewed Bridges

The abutment displacement coefficient R_A shall be used in the assessment of the effectiveness of the abutment.

$$R_A = \Delta_D / \Delta_{eff}$$

where:

Δ_D = The longitudinal displacement demand at the abutment from elastic analysis.

Δ_{eff} = The effective longitudinal abutment displacement at idealized yield.

If $R_A \leq 2$ The elastic response is dominated by the abutments. The abutment stiffness is large relative to the stiffness of the bents or piers. The column displacement demands generated by the linear elastic model can be used directly to determine the displacement demand and capacity assessment of the bents or piers

If $R_A \geq 4$ The elastic model is insensitive to the abutment stiffness. The abutment contribution to the overall bridge response is small and the abutments are insignificant to the longitudinal seismic performance. The bents and piers will sustain significant deformation. The effective abutment stiffness K_{eff} in the elastic model shall be reduced to a minimum residual stiffness K_{res}, and the elastic analysis shall be repeated for revised column displacements. The residual spring has no relevance to the actual stiffness provided by the failed backwall or diaphragm but should suppress unrealistic response modes associated with a completely released end condition.

$$K_{res} \approx 0.1 * K_{eff}$$

If $2 < R_A < 4$ The abutment stiffness in the elastic model shall be adjusted by interpolating effective abutment stiffness between K_{eff} and the residual stiffness K_{res} based on the R_A value. The elastic analysis shall be repeated to obtain revised column displacements.

7.8.2 Transverse Abutment Response

Seat type abutments are designed to resist transverse service load and moderate earthquake demands elastically. Typically seat abutments cannot be elastically designed to resist MCE demands because linear analysis cannot capture the inelastic response of the shear keys, wingwalls, or piles. The lateral capacity of seat abutments should not be considered effective for the MCE unless the designer can demonstrate the force-deflection characteristics and stiffness for each element that contributes to the transverse resistance.

The magnitude of the transverse abutment stiffness and the resulting displacement is most critical in the design of the adjacent bent, not the abutment itself. Reasonable transverse displacement of superstructure relative to the abutment seat can easily be accommodated without catastrophic consequences. A nominal transverse spring, K_{nom} equal to 50% of the transverse stiffness of the adjacent bent shall be used in the elastic demand assessment models. The nominal spring has no relevance to the actual residual stiffness provided by the failed shear key but should suppress unrealistic response modes associated with a completely released end condition. This approach is consistent with the stand-alone push analysis design of the adjacent bent and it is conservative since larger amounts of lateral resistance at the abutments that are not captured by the nominal spring will only reduce the transverse displacement demands at the bents. Any additional element, such as pile shafts (used for transverse ductility), shall be included in the transverse analysis with a characteristic force-deflection curve. The initial slope of the force-deflection curve shall be included in the elastic demand assessment model.

Diaphragm type abutments supported on standard piles surrounded by dense material can conservatively be estimated, ignoring the wingwalls, as 40 kips/in ($7.0\ \mathrm{kN}/\mathrm{mm}$) per pile.

7.8.3 Abutment Seat Width

Sufficient abutment seat width shall be available to accommodate the anticipated thermal movement, prestress shortening, creep, shrinkage, and the relative longitudinal earthquake displacement. The seat width normal to the centerline of bearing shall be calculated by equation 7.46 but not less than 30 inches (760 mm).

$$N_A \geq \begin{cases} \left(\Delta_{p/s} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_{eq} + 4\right) & \text{(in)} \\ \left(\Delta_{p/s} + \Delta_{cr+sh} + \Delta_{temp} + \Delta_{eq} + 100\right) & \text{(mm)} \end{cases} \tag{7.46}$$

N_A = Abutment seat width normal to the centerline of bearing

$\Delta_{p/s}$ = Displacement attributed to pre-stress shortening

Δ_{cr+sh} = Displacement attributed to creep and shrinkage

Δ_{temp} = Displacement attributed to thermal expansion and contraction

Δ_{eq} = The largest relative earthquake displacement between the superstructure and the abutment calculated by the global or stand-alone analysis

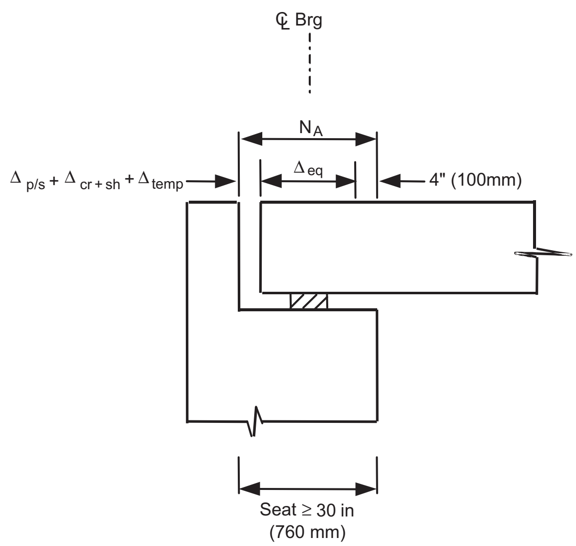

Figure 7.15 Abutment Seat Width Requirements

The "Seat Width" requirements due to the service load considerations (Caltrans Bridge Design Specifications and AASHTO requirements) shall also be met.

7.8.4 Abutment Shear Key Design

Typically abutment shear keys are expected to transmit the lateral shear forces generated by small earthquakes and service loads. Determining the earthquake force demand on shear keys is difficult. The forces generated with elastic demand assessment models should not be used to size the abutment shear keys. Shear key capacity for seat abutments shall be limited to the smaller of the following:

$$F_{sk} \leq \begin{cases} .75 \times \sum V_{pile} & \sum V_{pile} = \text{Sum of the lateral pile capacity} \\ 0.3 \times P_{dl}^{\sup} & P_{dl}^{\sup} = \text{Axial dead load reaction at the abutment} \end{cases} \tag{7.47}$$

Note that the shear keys for abutments supported on spread footings are only designed to $0.3P^{sup}_{dl}$.

Wide bridges may require internal shear keys to insure adequate lateral resistance is available for service load and moderate earthquakes. Internal shear keys should be avoided whenever possible because of maintenance problems associated with premature failure caused by binding due to the superstructure rotation or shortening.